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# Fatigue Verification of a Composite Railway Bridge Detail Based on Testing

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**Abstract**

The fatigue design practise for railway bridges according to European Eurocode is of general nature and developed to cover a large number of possible load and structure configurations. As the design procedure takes into account a wide range of parameters and uncertainties, usually it gives safe-sided results. Especially in case of residual life-time assessment of existing bridges but also in cases where the detail geometry is not in accordance with standard geometries from Eurocode, the code-based verification does not lead to satisfactory results. As a more accurate method a reality-based fatigue assessment is presented - including monitoring and testing. The main aspects of this method are demonstrated with an example: In the framework of a European research project fatigue tests on a composite bridge detail were performed. The investigated detail belongs to a 4-span twin girder composite railway bridge designed in 1994. The connection between main girder and diagonal bracing shows unfavorable characteristics in regard to fatigue resistance. It was therefore selected to conduct a fatigue verification based on testing. For this purpose 1:1 test specimens were designed and subjected to cyclic loading. The experimental results, analysis of the damage behaviour and comparison to common design practise are presented. The contribution gives a brief survey on the fatigue design according to European Eurocode - also highlighting its limitations. It is aimed at a better understanding of different approaches to fatigue verification. Emphasis is put on consequences of detailing and execution in regard to fatigue resistance.

*Keywords:* railway bridges, fatigue design, fatigue tests, bridge monitoring, Eurocode 1993-1-9

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## 1. FATIGUE DESIGN OF RAILWAY BRIDGES ACCORDING TO EUROCODE

### *1.1. General procedure*

The Eurocode-based fatigue design for railway bridges consists in general of the following steps:

a) Selection of a load model including appropriate parameters
b) Preparation of a global structural model
c) Determination of internal action effects (forces, moments)
d) Optional: preparation of a local member model, determination of action effects
e) Choice of the relevant fatigue detail, depending on member detailing
f) Comparison of stress-amplitude with fatigue detail resistance

It covers more or less all possible cases and applies modification factors for considering main structural and action properties. Normally the general approach leads to conservative assumptions when compared to real fatigue actions, in particular with regard to the applied loads.

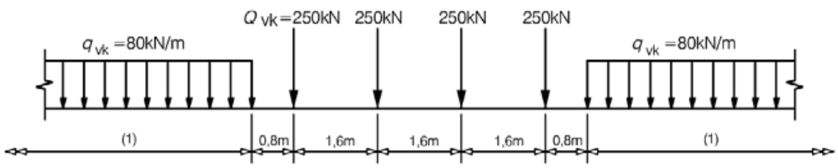


Figure 1: UIC load model 71 (CEN 2003).

The most common procedure is the use of the UIC load model 71 (Figure 1) for the determination of maximum stress amplitudes $\Delta\sigma_{71}$. Alternatively a specific combination of load models representing various train types can be used for the determination of stress-amplitudes instead of using UIC load model 71. However, this procedure requires the knowledge of the traffic occurring during the lifetime of the bridge and furthermore it needs to be approved by the railway operator. The general fatigue verification format for railway bridges is

$$\gamma_{Ff} \cdot \lambda \cdot \phi_2 \cdot \Delta\sigma_{71} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \tag{1}$$

with

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \leq \lambda_{\max} \tag{2}$$

| | |
|---|---|
| $\phi_2$ | specific dynamic increment for fatigue verification |
| $\Delta\sigma_{71}$ | stress-amplitude caused by the load model 71, relevant for the considered detail |
| $\Delta\sigma_c$ | stress reference values for fatigue resistance |
| $\gamma_{F,f}$, $\gamma_{M,f}$ | partial factors for fatigue verification |
| $\lambda$ | damage equivalence factor |
| $\lambda_1$ | span-length and influence line factor |
| $\lambda_2$ | traffic volume factor |
| $\lambda_3$ | design life factor |
| $\lambda_4$ | multi-track loading factor |
| $\lambda_{max}$ | 1.4 (limiting value) |

### *1.2. Limitations of the procedure*

The quality of this procedure depends very much on the accordance between the theoretical approach and the real loads and structural properties. Several discrepancies can be found:

a) Load model: the real traffic and loads may be rather different from the traffic used as a basis for the code development (usually the load model is a safe-sided assumption).

b) Global structural model: this model is decisive for the selection of the value and the dynamic increment (see equations above), also for the determination of the action effects within the structure.

c) Local member models: these models lead finally to the determination of local forces and stresses being relevant for fatigue. Effects like partial restraints or eccentricities may lead to a significant increase of stresses compared to stresses obtained by neglecting these effects.

d) Selection of fatigue details: the fatigue details provided by EC3-9 represent a large number of practical solutions. However, they cannot cover all eventualities found in the real structures.

e) Execution of the structure: in common practise there are often discrepancies between constructional design assumptions and the final workshop drawings and detailing from the manufacturer. Furthermore not removed secondary members, temporarily needed for erection purposes, have influence on the stress distribution within the fatigue detail.

Another very common difficulty is the assessment of the residual life-time of existing bridges. The following information are needed here:

- Damage accumulation due to already passed traffic
- Remaining fatigue resistance and life-time under expected future traffic

Such assessment procedures are often applied to important existing bridges where a replacement would be too expensive or which are not to be replaced for reasons of monument conservation. For such aged bridges the usual code-based verification does not lead to satisfying results. All these limitations of the Eurocode procedure show that a more accurate method based on reality can be useful.

## 2. FATIGUE ASSESSMENT BY MONITORING AND TESTING

### *2.1. General procedure*

The general procedure of reality-based fatigue assessment consists of the following principle steps:

a) Numerical investigation based on codes to determine relevant details and stresses. Identification of crucial details with regard to (possibly insufficient) fatigue resistance.

b) Bridge monitoring under ambient excitation. The results are used for verification and improvement of the global structural model.

c) Bridge monitoring under traffic (during a representative period). The results are used to assess the effects of traffic loads including bridge-train interaction.

d) Local Monitoring of relevant details

e) If required: fatigue testing of relevant details

### *2.2. Bridge monitoring*

Measurements of ambient bridge vibrations provide important information on the structural properties (stiffness, mass, damping). The results include effects like partial restraints, influence of ballast and rails, non-uniform mass distribution etc. Using the measured values an optimisation of the numerical model is possible, so that additional structural effects can be considered. Monitoring of bridges under traffic mainly aims at the determination of load effects under real conditions of passing trains. Although trains from the past can not be measured, a realistic estimation of the damage accumulation can be performed. It is also possible to extrapolate the results to future loads from expected traffic. With these information an appropriate load model is defined, which includes relevant train types and also dynamic effects. Finally - in cases where critical details are reliably identified - a monitoring of specific bridge parts or even details leads to an accurate assessment of structural effects. Here a combined approach of strain and frequency measurements appears to be the best choice allowing both the determination of real fatigue stresses and the detection of damages. Extensive information about bridge monitoring and its application areas are given in (Wenzel 2009).

### *2.3. Fatigue testing*

For critical details which are not directly comparable to detail categories provided by the codes fatigue testing may be necessary to verify the real fatigue resistance of these details. It is recommended to design the test set-up in such a way that effects observed in the real structure are taken into account - e.g. structural boundary conditions and eccentricities. Cross-sections and materials as well as the quality of execution (e.g. for welds) shall be comparable to the properties of the detail in the real structure.

## 3. EXAMPLE: MELK BRIDGE, AUSTRIA

### *3.1. Introduction*

At the Austrian “Melk Bridge” - which was case study in the DETAILS project (European Commission 2010) - a composite bridge detail possibly being prone to fatigue loading was found, cf. Figure 2. In order to verify the applicability of combined analysis-monitoring-testing procedure this detail was selected to investigate its fatigue resistance.

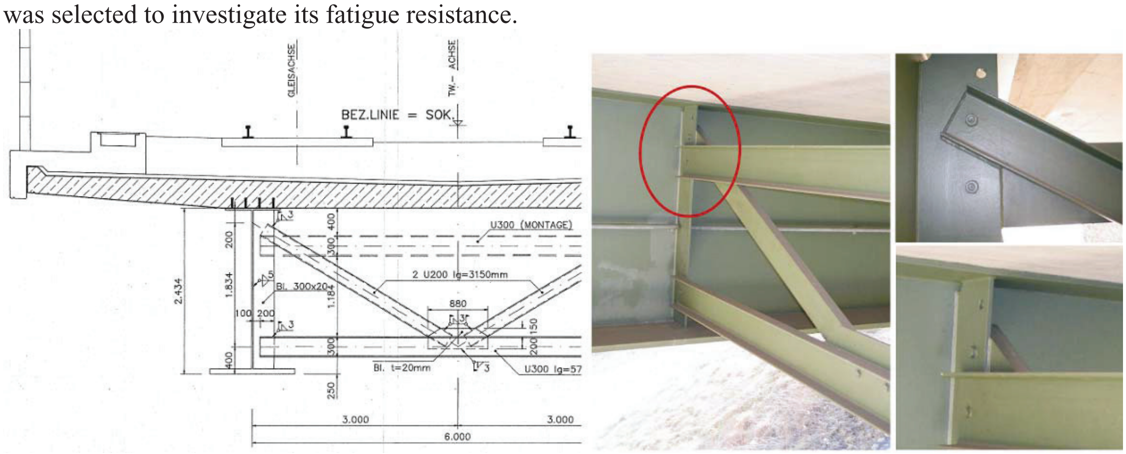


Figure 2: Selected detail: welded connection diagonal bracing / main girder stiffener.

Although forces and stresses induced into this detail by passing trains are not critical in terms of fatigue damage - which was verified by monitoring and assessment of internal forces within the bracing sub system - the example represents typical aspects of uncertainties with regard to fatigue design and execution. Part of the investigation was the comparison between the common code-based verification procedure and the approximately real situation on site.

*3.2. Fatigue design based on codes and simplified structural models*

Usually the code-based verification procedure starts with the determination of internal forces from the global model. In the following the internal forces (e.g. torsional moment) are applied to the structural sub model of the transverse bracing system in order to obtain relevant member forces. In general this calculation is done by simplified modelling assuming clear hinged or rigid connections. Here a calculation of member forces was not required, because they were already identified by means of monitoring. As the temporary "Montage" beam is assumed to be removed after erection, it would not be considered within the calculation. Rather often also the eccentricities are neglected and the welded connections are assumed to be pinned.

The next step is the selection of the appropriate detail category according to the details provided by the codes. The selection of the detail category is based on design assumptions, cf. Figure 3. This selection leads to a detail category of 45*. This detail category, however, is only valid for welds which stop at a minimum distance of 10 mm from the edge (CEN 2005a).

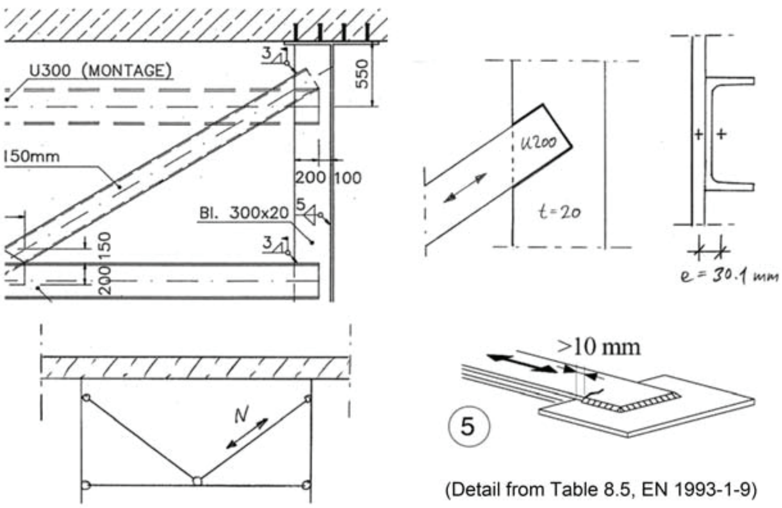


Figure 3: Extract of workshop drawings, internal force in diagonal bracing, eccentricity and classification due to Eurocode.

The simplest method for verification is done using the axial force only for calculation of the stress amplitude:

$$\Delta\sigma = \frac{\Delta N}{A} \tag{3}$$

With a unit force of N = 100 kN this leads to a stress amplitude of

$$\Delta\sigma = \frac{100}{32.2} = 3.11\frac{kN}{cm^2} \tag{4}$$

A more accurate stress amplitude is determined by taking the eccentricity into account:

$$\Delta\sigma = \frac{\Delta N}{A} + \frac{\Delta N \cdot e}{W_z} \tag{5}$$

With a unit force of N = 100 kN this leads to a stress amplitude of

$$\Delta\sigma = \frac{100}{32.2} + \frac{100 \cdot 3.01}{73.63} = 7.19 \frac{kN}{cm^2} \tag{6}$$

The eccentricity between the attached U200 and the plate t = 20 mm leads to an increase of stresses by a factor of more than 2.

### *3.3. Real situation*

A check of the detail "as executed" showed the real situation on site, cf. Figure 2. With regard to the assumptions made in chapter 3.2 the following aspects have to be pointed out:

a) The weld does not stop 10 mm before the edge of the plate; it is a "weld all around"

b) The weld is present also at the opposite side (along the edge of the plate).

c) The temporary beam has not been removed; moreover this beam has been also weld to the vertical plate resulting in a rather complex detail.

Furthermore the welded connections are obviously not pinned. Thus frame actions and additional moments can be expected. The apparently valid detail category is detail 6 from Table 8.5, EN 1993-1-9 (CEN 2005a) leading to $\Delta\sigma_c^* = 56$ N/mm² ($t_c$<20). The reality differs considerably from the initial design - with significant influence on the fatigue resistance of the detail.

### *3.4. Fatigue verification by testing*

In order to check if the assumptions made on the basis of detail categories, provided by the code, are valid for the real detail, fatigue tests on 1:1 test specimen were performed, cf. Figure 4.

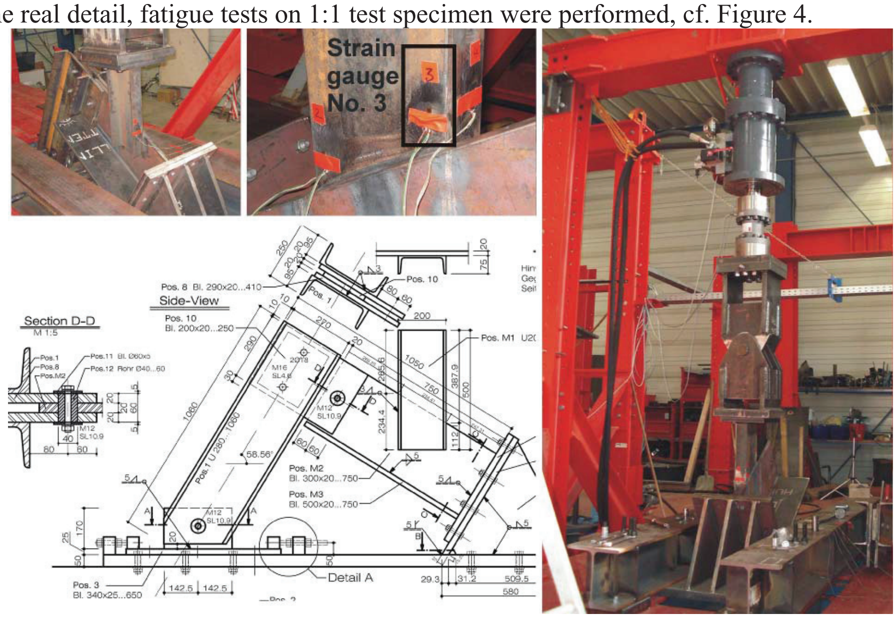


Figure 4: Test setup related to fatigue tests on Melk detail.

The test specimens were designed similarly to the real situation except for the temporary beam which was not considered. The rigid connection of the main girder to the bridge deck - in reality a connection with shear studs - was achieved by a bolted connection to a stiff sub structure. In order to simulate the influence of frame action, the load was introduced eccentrically. The real stress distribution was measured by strain-gauges placed at significant positions on the detail. All tested specimens showed the same damage behaviour, cf. Figure 5. An initial crack started at the edge near to strain-gauge number three (marked in Figure 4 with a black box). Then the crack continued rapidly until the U-profile was damaged entirely.

Figure 5: Damage behaviour of test specimens.

The results of the tests have been evaluated applying the following methods:

a) $\Delta\sigma = N/A$

b) $\Delta\sigma = N/A + (N \cdot e_z)/W_z$

c) $\Delta\sigma$ = measured maximum stress

where methods a and b correspond to the simplified calculated stresses and method c considers all eccentricities and local structural discontinuities, leading to a non-uniform stress distribution.

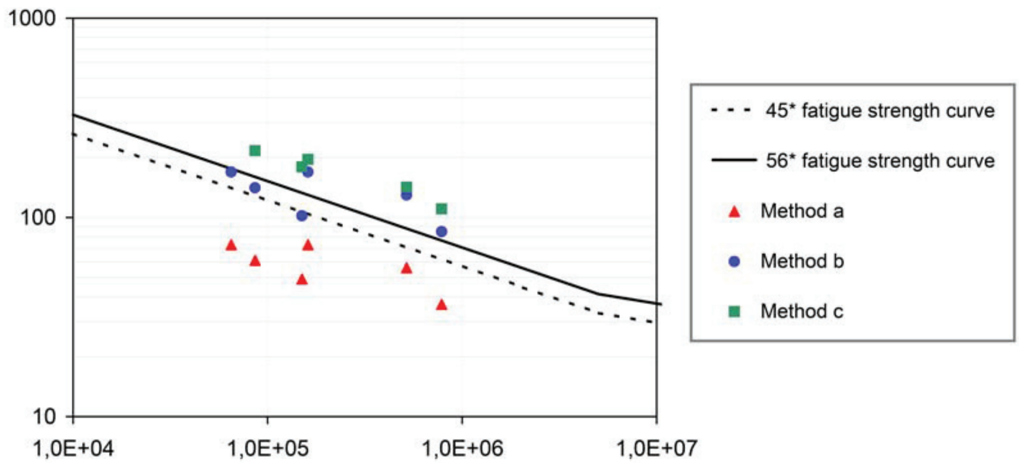


Figure 6: Fatigue test results (each specimen represented by three points); methods a, b, c.

The test results are given in an S-N-diagram and compared to the nominal S-N-curves with $\Delta\sigma_c* = 45$ N/mm² and $\Delta\sigma_c* = 56$ N/mm², cf. Figure 6. These results show very clearly that a simple consideration only of the axial force (method a) or of the axial force with the eccentricity related to the weak axis (method b) leads to an underestimation of fatigue action effects (stresses), which in case a) are considerably lower compared to the referenced curves from Eurocode. That means, that a simplified calculation of design-relevant stress-amplitudes $\Delta\sigma_{71}$ (cf. equation 1) by using method a or b does not give safe-sided values being applicable for fatigue design of this detail. Only the consideration of real stresses (method c) leads to safe-sided values. The next step to an improved fatigue design is the calculation of a detail-specific $\Delta\sigma_c$-value from the tests. Consequently the fatigue verification according to equation (1) has then to be performed in consideration of real stresses.

## 4. CONCLUSIONS

The Eurocode based fatigue verification for railway bridges in steel is a strong tool covering a large spectrum of applications. But in cases, where this standard procedure does not lead to satisfying results - due to limitations of the design methods - a different approach based on testing can be applied, even though this method is much more extensive.

To achieve optimum results the approach requires

- knowledge of realistic design-relevant stress-amplitudes (e.g. by means of monitoring)
- knowledge of relevant structural boundary conditions

The fatigue verification can then be performed in

- consideration of realistic actions (derived from monitoring data)
- consideration of realistic resistances (derived from experimental fatigue tests)

An adequate bridge detail was chosen to demonstrate this approach which is not directly comparable to standard details given in Eurocode. Furthermore the influence of eccentricities and local structural discontinuities on design stress amplitudes is hard to predict. The example reveals the influence of structural detailing and methods of stress determination on the fatigue verification.

## ACKNOWLEDGMENTS

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